ASCE 4-16 STANDARD-BASED PROBABILISTIC SEISMIC SSI ANALYSIS FOR DESIGN-BASIS AND FRAGILITY ANALYSIS

D. M. Ghiocel, *Ghiocel Predictive Technologies, Inc., New York, US*

ABSTRACT

Probabilistic soil-structure interaction analysis (PSSIA) is capable of capturing in much more detail the uncertainties related to the seismic motion, soil layering and structural behaviour than deterministic SSI analysis (DSSIA). In the new ASCE 4-16 standard it is stated that the purpose of the analytical methods included in the standard is to provide reasonable levels of conservatism of structural design to account for seismic analysis uncertainties. The paper illustrates the application of the new ASCE 4-16 standard recommendations for probabilistic SSI analysis for the design-basis level (DBE) and the beyond design-basis level (BDBE) applications including fragility computations. The ACS SASSI with Options PRO and NON is used in this paper.

1. PROBABILISTIC MODELING

The ASCE 4-16 standard Section 5.5 recommends for probabilistic SSI analysis the stochastic simulation using the Latin Hypercube Sampling (LHS). The ASCE 4-16 standard addresses both the probabilistic site response analysis (PSRA) and the probabilistic SSI analyses (PSSIA) in Sections 2 and 5.5, respectively.

Probabilistic modelling should include at least the random variations due to:
- Response spectral shape model for the seismic input
- Low-strain soil shear wave velocity Vs and hysteretic damping D profiles for each soil layer
- Soil layer shear modulus G and hysteretic damping D as random functions of soil shear strain
- Equivalent linear/effective stiffness and damping for concrete structural elements depending on stress/strain levels in different parts of the structure

The ACS SASSI Option PRO modules include the above probabilistic modelling aspects following the ASCE 4-16 standard recommendations for both PSRA and PSSIA (Ghiocel, 2017). Figure 1 shows a generic chart of the Option PRO PSSIA simulations.

For the probabilistic SSI response simulations, the input is represented as an ensemble of randomized seismic input motion sets. Each set consists of two horizontal components and one vertical component.

The seismic motion spectral amplitude is assumed to be a lognormally distributed random variable or vector/field. Option PRO includes two probabilistic simulation methods for generating input acceleration time histories that are recommended in the ASCE 4-16 standard Section 5.5 (Figure 2):

1) Method 1 that assumes that spectral shape is deterministic, constant shape curve, and

2) Method 2 that assumes that spectral shape is a random, variable shape curve.

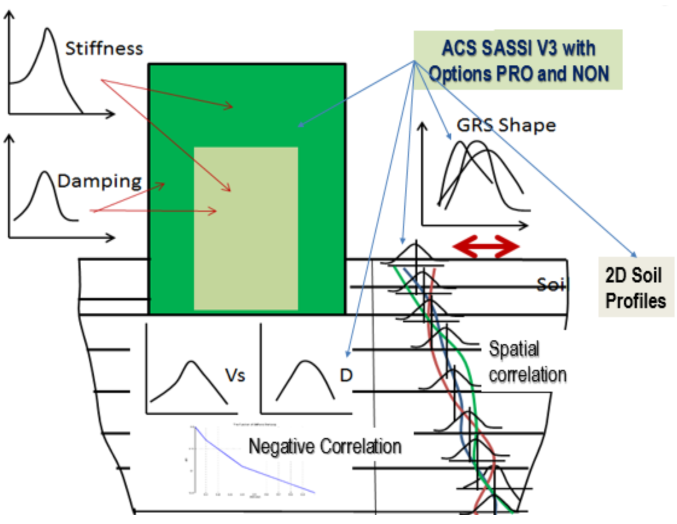


Figure 1 Probabilistic Seismic SSI Analysis Chart

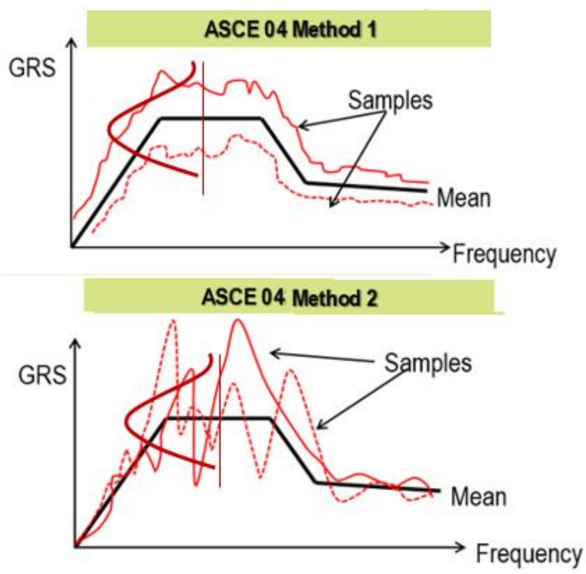


Figure 2 Probabilistic Seismic GRS Models

The low-strain Vs and D per soil layers are assumed to be statistically dependent lognormal or normal random variables. The statistical dependence is due to their joint dependence on the soil shear strain in each layer.

In Option PRO, there are several options implemented to address the Vs and D statistical dependence. Each geological layer including several computational soil layers can be defined with different statistics, as means, coefficients of variation and correlation lengths. Thus, in general, the soil profiles are made of several segments for which the soil profile spatial correlation with depth is assumed to be constant. The soil profile simulations are based on the probability transformed-space Karhunen-Loeve expansion models (Figure 3).

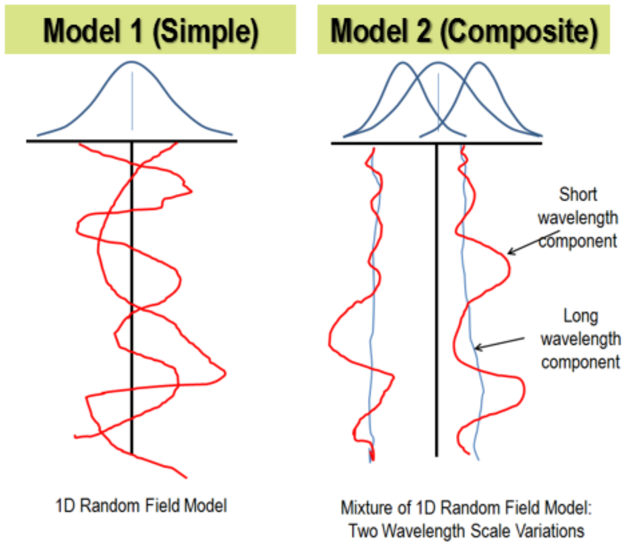


Figure 3 Probabilistic Soil Property Profile Models

The soil Vs and D profiles are assumed to be either i) Model 1, 1D random field with a spatial correlation structure with depth, or ii) Model 2, a random field mixture of a short-wavelength 1D component and a large-wavelength 1D component. The first modelling option produces ergodic field samples, while the second modelling option produces non-ergodic field samples since it contains two sources of uncertainties.

The Model 2 was also recommended by the Princeton university researchers based on various soil field measurements (Popescu, 1996). The selection of the soil profile model should be made based on the Vs field measurements on the site.

The soil shear modulus G and damping D curves as functions of the soil shear strain in each layer, are modelled as 1D random field models with slow-variations or large wavelength in the shear strain space (Figure 4).

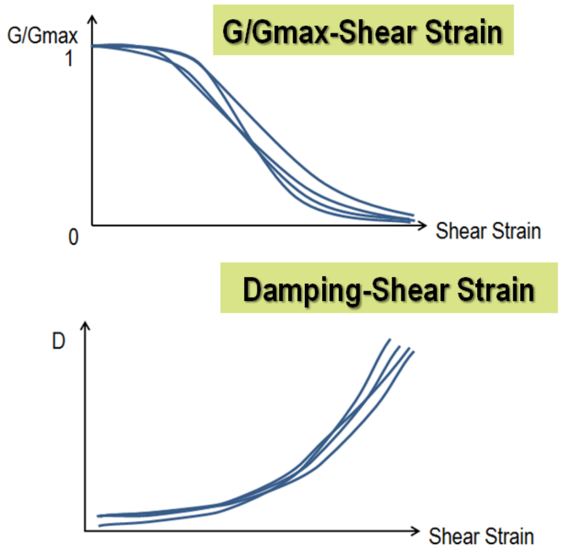


Figure 4 Probabilistic Soil Material Curves

For structural modelling, the effective stiffness and damping values in the concrete walls should be defined as separated random variables for different parts of the structure which have different local stress/strain levels. Defining the effective stiffness and damping for each wall could be a labor intensive expert activity since for each seismic simulation, the stress/strain levels in the structure may vary substantially. The ACS SASSI Option NON can be used to automatically compute the effective stiffness and damping in the concrete walls as function of the local shear or bending strain.

The ACS SASSI Option PRO includes a number of seven probabilistic modules that generate the LHS randomized samples for the PSSIA input. These probabilistic modules include the ProEQUAKE, ProSOIL, ProSITE, ProHOUSE, ProMOTION, ProNON, ProSTRESS and ProRESPONSE modules.

A probabilistic analysis, either PSRA or PSSIA, there are three distinct steps to be completed:

1) Generate an ensemble of simulated probabilistic input files using LHS with the Option PRO pre-processing modules,

2) Run the ensemble of LHS sample input filesto compute the corresponding SSI response files with the ACS SASSI main software modules, and

3) Post-process statistically the ensemble of the LHS responses with the Option PRO post-processing modules.

2. DESIGN-BASIS (DBE) APPLICATION

ASCE 4-16 based probabilistic and deterministic SSI analyses were comparatively performed for a deeply embedded SMR SSI model. The probabilistic SSI analyses assumed that the spectral shape of the site-specific ground response spectra, the soil stiffness and damping profiles were idealized as random fields. The structural stiffness and damping random variations were modelled as a pair of correlated random variables that depend on the computed structural stress levels.

The comparative SSI results include in-structure response spectra (ISRS) at different locations. The probabilistic SSI analysis results for the mean ISRS and 84% NEP ISRS (slightly higher than the 80% NEP ISRS) are compared with the deterministic SSI analysis envelope ISRS computed including the three deterministic soil profile variations, namely, lower bound (LB), best-estimate (BE) and upper bond (UB).

Figure 5 shows the a deeply embedded generic SMR SSI model. The SMR structure has a size of 200ft x 100ft x 100ft (H x L x W) with a embedment of 140ft depth (Ghiocel, 2017).

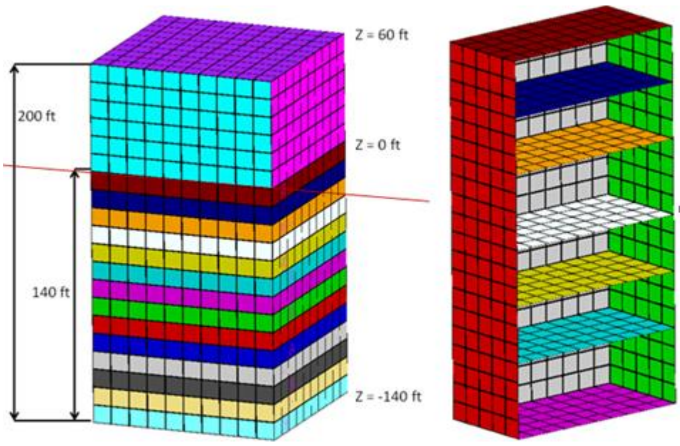


Figure 5 140 ft Embedded SMR SSI Model

The soil profile shown in Figure 6 is highly non-uniform with a soil layer stiffness variation inversion within the embedment depth. The seismic motion was input at the SMR foundation level (FIRS) at the 140 ft depth (elevation 0 ft).

For probabilistic analyses, the in-column FIRS input motions were computed based on the probabilistic site response analysis using 60 LHS samples. The 60 randomized soil profiles are plotted in Figure 6. For statistical segments were considered for soil profile modelling. Both the Model 1 and the Model 2 were comparatively used for the soil profile probabilistic modelling. The statistics of the Vs and D soil profiles have variation coefficients of 20% and 30%, respectively. The Vs and D statistical dependence as function of soil shear strain level is captured by a coefficient of correlation of -0.40. The smoothness of the soil variation profiles was controlled by the correlation length parameters that vary with depth (for the four segments). The correlation lengths were considered 40 ft for the segments 1 and 3 down from the ground surface and 100 ft for the segments 2 and 4. These correlation length values were used for both the Model 1 and the Model 2 short-wavelength component variations. As shown in Figure 6, the probabilistic seismic input was defined by the outcrop UHRS motion simulated at the 500 ft depth bedrock

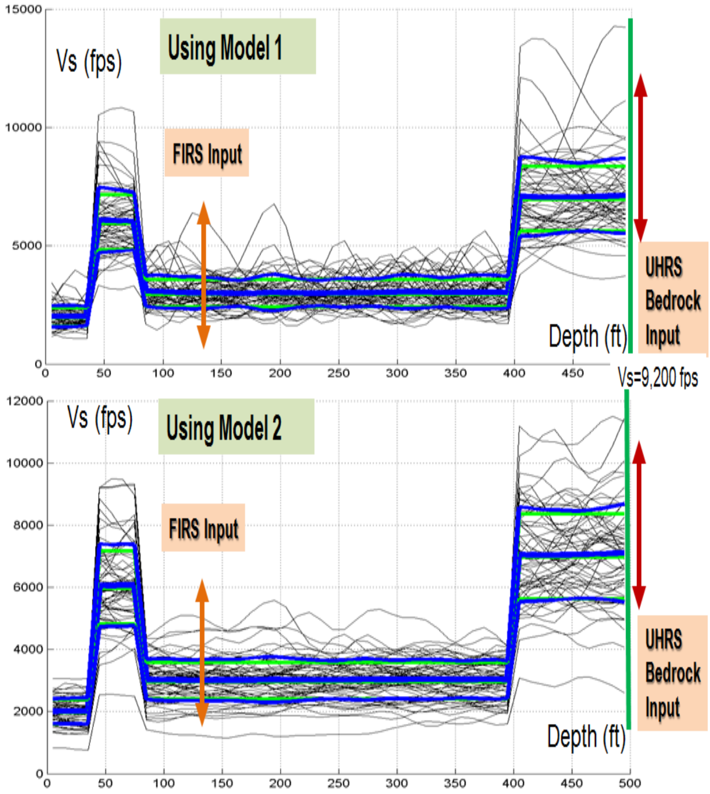


Figure 6 Vs Soil Profile Simulations; Random Samples (black), Simulated Statistical Curves (blue) And Target Statistical Curves (green)

The 60 simulations of the in-column FIRS obtained using PSRA are were further used for the PSSIA of the SMR structure. Probabilistic SSI ISRS results are shown in Figures 7 through 9. Each figure includes comparative ISRS probabilistic 84% NEP ISRS vs. deterministic ISRS for the two profile models, Model 1 and Model 2.

It should be noted that the deterministic seismic input is defined by the in-column FIRS computed for the LB, BE and UB soils based on the (probabilistic) mean outcrop UHRS FIRS obtained for the 60 probabilistic site response simulations. Thus, the deterministic SSI analysis input is not based on the Design Response Spectra (DRS) FIRS input that should be used in the "conventional" deterministic design-basis seismic SSI analysis that is a combination of the mean Uniform Hazard Response Spectra (UHRS) FIRS computed for two annual seismic hazard probabilities, specifically, 1.e-4 and 1.e-5, as described in the ASCE 43-05 standard, and more recently in the new ASCE 43-18 standard draft.

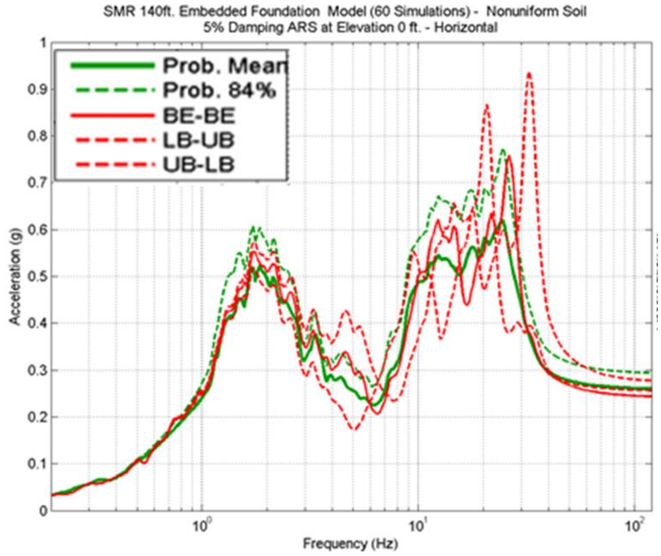
It should be noted that the comparison of probabilistic SSI analysis and “conventional” deterministic SSI analysis results depends on the seismic hazard level considered for the probabilistic simulations. If the 1.e-4 annual seismic hazard probability is considered, then, the “conventional” deterministic SSI results based on DRS input, should be increased by the ratio between ratio DRS and 1.e-4 mean UHRS, while if the 1.e-5 annual seismic hazard probability is considered, the “conventional” deterministic SSI results should be decreased by the ratio between 1.e-5 UHRS and DRS.

To avoid any confusion, the deterministic SSI seismic input is defined by the same (probabilistic) mean UHRS FIRS computed from the probabilistic site response simulations, and not by the “conventional” deterministic DRS FIRS.

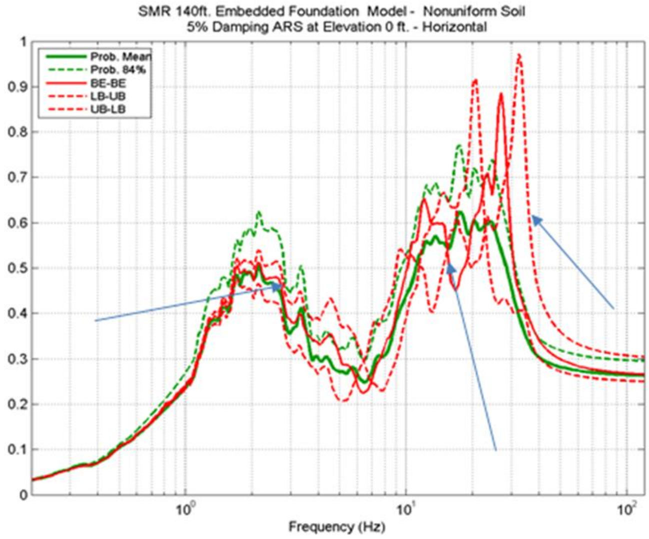
For probabilistic structural modeling, it was considered that the entire SMR model has the same effective stiffness and damping modeled as lognormal random variables. The effective stiffness was assumed with a mean of 0.80 of the elastic stiffness and a c.o.v of 10%, while the effective damping was assumed with a mean of 6% and a c.o.v. of 30%. The statistical dependence between the two variables was included by a negative correlation coefficient of -0.80. For deterministic structure modeling the uncracked stiffness and 4% damping were assumed.

Figures 7 thru 9 show comparisons of the deterministic ISRS for the LB, BE and UB soils, (red lines, with solid line for BE soil) and the probabilistic mean and 84% NEP ISRS (green lines, solid line for mean values) at 0ft elevation (basemat level) and 170ft elevation (30ft above ground level). The left side plots of the Figures 7 through 9 show the ISRS results based on the Model 1 soil profile (Figure 6, upper plot), while the right side plots show results based on the Model 2 soil profile (Figure 6, lower plot). It should be noted that the probabilistic model used for the simulation of the soil profiles affects slightly both the probabilistic and the deterministic ISRS. The deterministic soil profiles of Vs and D correspond to the 16%, mean, and 84% NEP statistical estimates, and, therefore, they are affected by the probabilistic soil profile modelling using Model 1 or Model 2.

Figures 7 and 8 show the ISRS computed at 0ft elevation (basemat) for the horizontal direction and vertical direction, respectively. The blue arrows on the plots indicate frequency bands for which some discrepancy between the probabilistic 84% NEP ISRS and the deterministic envelope ISRS are noticed.



a) Using Model 1 Soil Profile

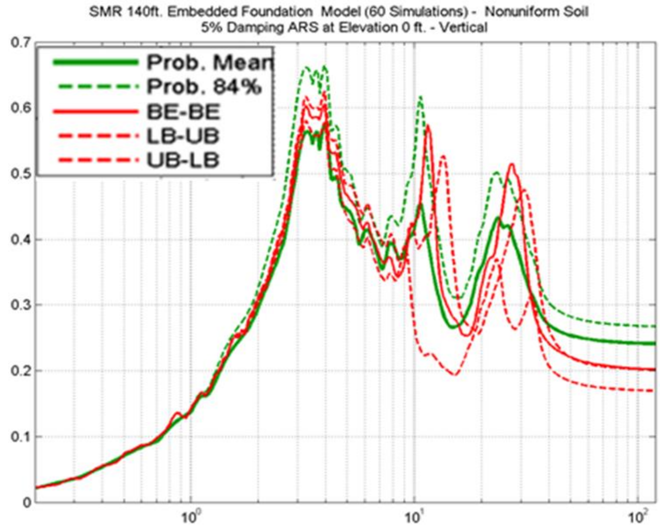


b) Using Model 2 Soil Profile

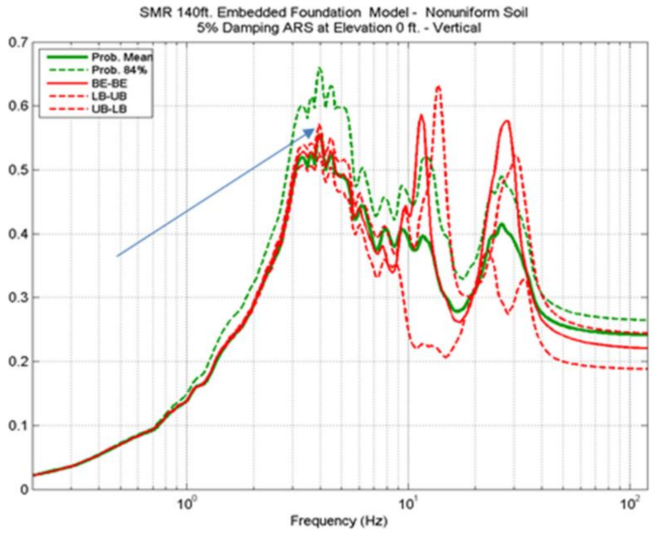
Figure 7 Horizontal ISRS at the Basemat Level

For the horizontal direction, as shown in Figure 7, at lower frequencies the probabilistic 84% NEP ISRS are slightly larger, up to 20% for Model 2, while in the high-frequency range, above 10 Hz, the deterministic ISRS, especially for the UB soil, has a much larger peak amplitude than the probabilistic 84% NEP ISRS, possibly with a 100% increase or even more. For the vertical direction, as shown in Figure 8, similar trends are noticed.

Figure 7 ISRS plots also indicate that the deterministic soil profiles, LB, BE and UB, produce a highly amplified SSI analysis responses in the 20-30 Hz range, since these profiles are basically outside of the range soil profile random variations. This is due the fact that producing a randomized soil profile that is similar to the deterministic soil profiles for which all soil layers are being simultaneously stiffer or softer, has a very low likelihood, or in other words, the deterministic soil profile corresponds to a small occurrence probability within the random sample space. In deterministic SSI analysis, the soil profiles have an implicit occurrence probability of unity since it corresponds to a sure event.



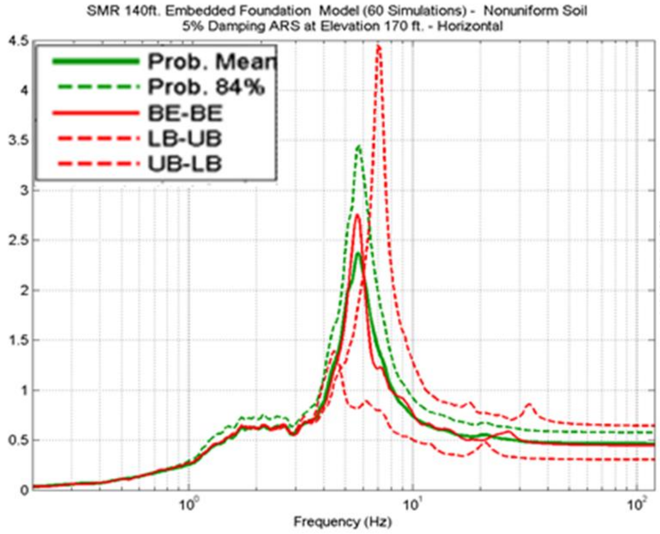
a) Using Model 1 Soil Profile



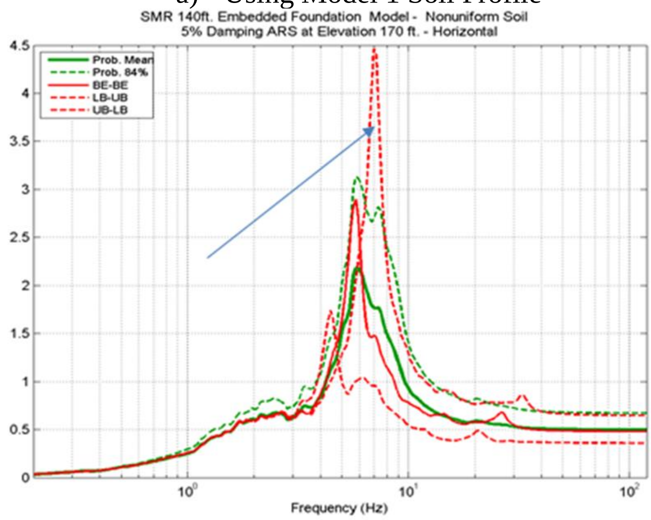
b) Using Model 2 Soil Profile

Figure 8 Vertical ISRS at the Basemat Level

Figure 9 shows the horizontal ISRS computed at 170ft elevation (30ft above ground level). The deterministic envelope ISRS spectral peak, in fact only for the UB soil, is much larger than the probabilistic 84% NEP ISRS, about 50% larger. This significant difference between the deterministic and probabilistic ISRS is mainly due to the 4% low damping value used for the deterministic SSI analysis for the uncracked concrete that is lower than the randomized damping values assumed with a statistical mean of 6% for the probabilistic SSI analysis. It should be also noted that the deterministic envelope ISRS peak is at 7 Hz, while the probabilistic 84% NEP ISRS peak is at 5.5 Hz that is about 20% lower than the deterministic envelope ISRS peak. Also, the LB soil ISRS peak occurs at a frequency of 4.3 Hz, the BE soil ISRS peak occurs at 5.5 Hz, and the UB soil ISRS peak occurs at 7.0 Hz. The peak frequency shifts is about +/- 20% around the BE soil ISRS peak frequency.



a) Using Model 1 Soil Profile



a) Using Model 2 Soil Profile

Figure 9 Horizontal ISRS at 30ft Above Ground Level

The large horizontal deterministic ISRS peaks noticed in Figures 7 and 9 could appear to be a penalty of the deterministic SSI analysis on the economical aspects of the nuclear design.

There is a pressing need for having more comparative probabilistic-deterministic investigations based on the ASCE 4-16 standard recommendations for various case studies with different complex FE models, not only sticks, to be able to understand in all details the differences between the results of the probabilistic SSI versus deterministic SSI. In concept, probabilistic SSI is superior to deterministic SSI that trends to become overly conservative for the high elevation levels, but not always (!).

3. BEYOND DESIGN-BASIS (BDBE)

Probabilistic SSI analyses for the beyond design-basis (BDBE) applications are typically performed for seismic input review levels that are much larger than the design-basis (DBE) seismic input, often by 2-3 times. For such much larger BDBE seismic inputs, the role of the nonlinear soil and structure behaviours become very important SSI modelling aspects for obtaining meaningful seismic margin results. Herein, the application of the probabilistic SSI analysis per the new ASCE 4-16 standard is presented in the context of the seismic fragility analyses.

The new ASCE 4-16 standard provides a probabilistic physics-based modelling framework for computing seismic SSI response variations that is a good basis for adequately deriving the fragility analysis data, and by this to substantially reduce the "expert" subjectivity. The "expert" subjectivity", in many situations, can introduce significant, artificial biases in the SSC seismic margin evaluations which can be sometimes too crude, or even inappropriate on a case-by-case basis.

An accurate seismic fragility analysis should include several seismic hazard levels or review levels, not only a single seismic hazard level or review level. At the least, three seismic hazard levels or review levels should be considered as acceptable. For performing a pertinent probabilistic SSI analysis per the new ASCE 4-16 recommendations, probabilistic models for the seismic input motion, the soil profile and the structure should be defined for each seismic hazard level, as briefly described below:

- Probabilistic outcrop UHRS SEISMIC input at the bedrock should be defined for each review level. Usually, at the least three levels, 1e-4, 1e-5 and 1e-6 should be considered for the seismic hazard annual probability. Based the PSHA studies, the deaggregated bedrock UHRS inputs should be defined for the governing seismic events as functions of the magnitude and epicenter distance. Probabilistic models should assume that the bedrock UHRS frequency content has random variations.

- Probabilistic SOIL layer profiles should be defined for each review level are necessary. Probabilistic models should include at the least the shear velocity Vs, and damping D soil profiles and the material constitutive curves applicable for the soil condition sites. For the soil sites, the nonlinear soil behavior should be included for each review level and each seismic input simulation.

- Probabilistic STRUCTURE material properties, namely, the effective stiffness and damping, should be defined for each review level. Probabilistic models should include the effective stiffness and damping variations. Since the effective/equivalent-linear stiffness and damping properties are functions of the stress/strain level, their values are different for different parts of the structure. The nonlinear structure behavior should be included for each review level and each seismic input simulation.

The seismic hazard curve considered for the investigated case study that is for a rock site is shown in Figure 10. The maximum ground acceleration levels vary from 0.10g to 1.60g. The DBE level corresponds to the outcrop UHRS defined for a 0.25g maximum ground acceleration in the horizontal direction.

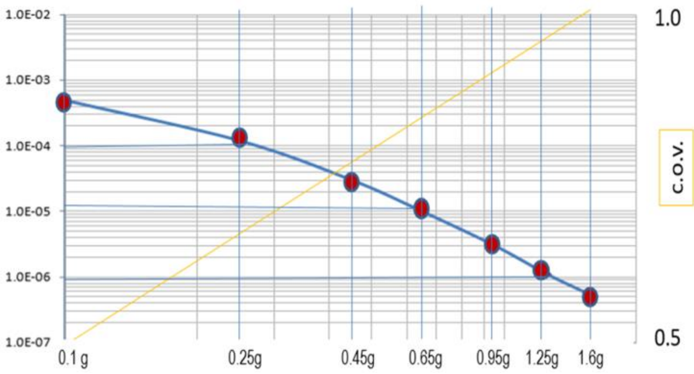


Figure 10 Seismic Hazard Curve for Rock Site

The seismic hazard curve is used to define seven seismic hazard or review levels which correspond to the annual mean occurrence probabilities of 3.2.e-4, 1.0e-4, 3.2e-5, 1.0e-5, 3.2e-6, 1.0e-6 and 3.2e-7. The c.o.v. values associated to the seismic hazard curve are plotted by the yellow-brown line and vary from 50% at 0.10g to 100% at 1.60g.

To compute the total seismic risk or failure probability curves for SSCs, the seismic hazard curve slope should be "convolved" with the SSC fragility curves as illustrated in Figure 11. The overall predicted risk including all seismic hazard events is finally obtained by integrating the total risk curve over the entire ground acceleration axis. It should be noted that the seismic hazard and fragility curves are affected by the epistemic or modelling uncertainties. These uncertainties are illustrated in Figure 11 by the probability density functions (blue colour areas) and their random samples (black lines). These uncertainties are further propagating to the total risk curves and, finally, to the overall predicted risk, Pf, affecting its confidence interval. The overall predicted risk should be determined with high confidence levels, since usually the overall risk estimate has large uncertain variations.

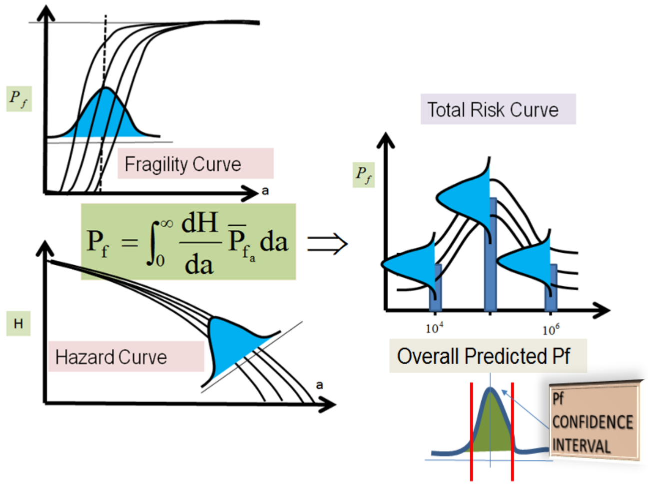


Figure 11 Seismic Risk Prediction Chart

The epistemic or modeling uncertainties were computed using the probabilistic seismic response variations for two sets of probabilistic SSI analysis, as follows:

1) Probabilistic SSI analysis including total or composite probabilistic variations composed by the superposition of the randomness variations and the epistemic or modeling uncertainty variations;

2) Probabilistic SSI analysis including only the random variations (with no epistemic uncertainties).

The epistemic or modeling uncertainty variations can be computed by subtracting the random variations from the total or composite variations of the seismic responses. The simulation procedure permits an accurate evaluation of the epistemic uncertainty variations based on computing the epistemic uncertainty variates for each pair of the 60 probabilistic response simulations produced in steps 1) and 2).

The probabilistic SSI analysis was performed for each of the seven selected seismic hazard or review levels using the ACS SASSI Option PRO software. Figure 12 shows the 60 probabilistic outcrop UHRS input simulations which were used for the PSRA and the PSSIA computations for the 0.25g (design-basis) seismic hazard level input.

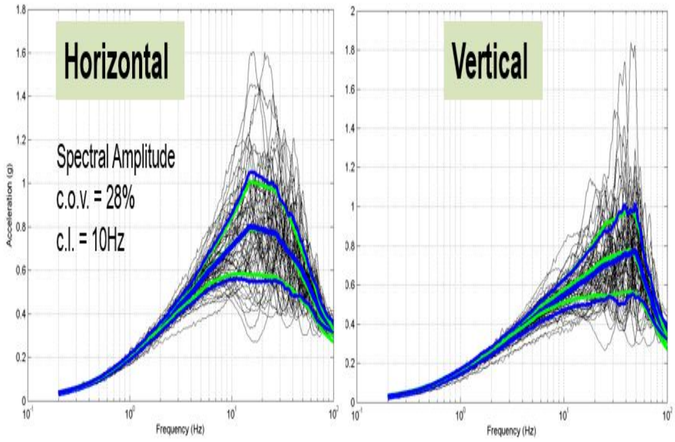


Figure 12 Probabilistic Horizontal and Vertical UHRS Simulations for 0.25g Seismic Input Level

For each UHRS simulation, a spectrum compatible acceleration time history was generated. Since the investigated site is a rock site, the nonlinear soil behavior effects are negligible. For random variations, the UHRS amplitude was assumed as a lognormal random variable with the mean equal to mean UHRS and a c.o.v. of 28%. The ASCE 4-16 Section 5.5 Method 2 with randomized UHRS spectral shapes was applied (ProEQUAKE module). The correlation length was taken 10 Hz. To include the epistemic uncertainties a lognormal random factor with mean of 1.0 and c.o.v. of 25% was applied.

The 60 soil profile simulations are shown in Figure 13.

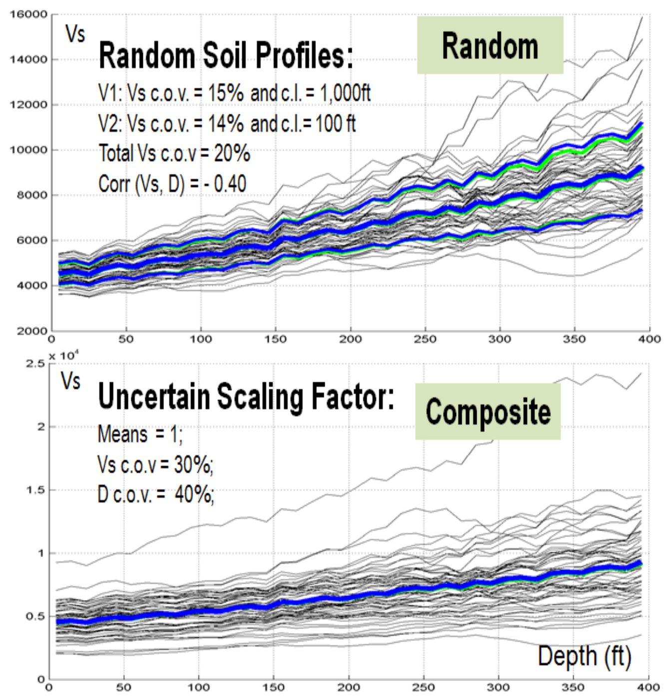


Figure 13 The Vs Soil Profile Probabilistic Simulations

For the randomness variations, the soil profile probabilistic modelling was based on the Model 2 that includes a mixture of two lognormal random field components, with a short wavelength and a long wavelength, respectively. The c.o.v. of the Vs and D soil profiles were 20%, with a c.o.v. of 14% for the short wavelength component and a c.o.v. of 15% for the long wavelength component. The correlation length was 100 ft for short wavelength component and 1000 ft for long wavelength component. The statistical dependence between Vs and D was simulated by a correlation coefficient of -0.40. For the epistemic uncertainty variations of Vs and D profiles, lognormal random scale factors with a mean of 1.0 and c.o.v. of 30% and 40%, respectively, were considered.

The investigated concrete shearwall nuclear building used for probabilistic SSI analysis is shown in Figure 14. For the seismic inputs associated with seismic hazard levels which are well beyond the 0.25g design-basis level, up to 1.60g, the nonlinear structure behaviour is an important aspect of the SSI modelling. For the probabilistic nonlinear SSI analysis, the ACS SASSI Options NON and PRO modules were combined.

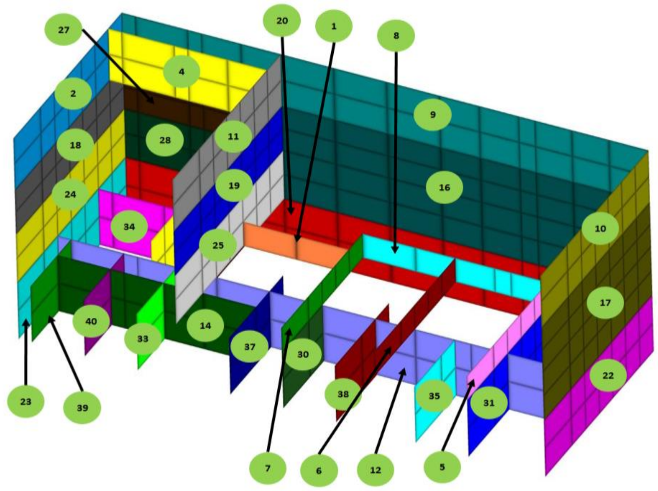


Figure 14 Low-Rise Shearwall Concrete Building Nonlinear Model

To include the nonlinear concrete behaviour, the structure model has to be split in a number of wall "panels". The wall panel is part of the concrete shearwall that is under a relative uniform shear or bending deformation. Herein, the investigated nuclear building nonlinear model included 40 wall panels, as shown in Figure 14 (Ghiocel and Saremi, 2017).

For each nonlinear wall panel, a back-bone curve (BBC) has to be defined. The BBC curves should have a smooth shape and variation that describes the nonlinear behaviour of the wall panels under the lateral seismic loading. The smoothed BBC were automatically generated based on the input data on the cracking and ultimate capacity shear force values and assuming that the secant cracked stiffness between the cracking and yielding points is half of the uncracked stiffness as recommended in the ASCE 4-16 (Ghiocel and Saremi, 2017).

The BBC were assumed to vary randomly assuming a lognormal random factor with mean of 1.0 and the c.o.v. of 15% for random variations and a c.o.v. of 33.5% for composite variations. For the nonlinear hysteretic behavior the Cheng-Mertz shear (CMS) hysteretic model was used (Ghiocel, 2015). This CMS hysteretic model is a part of the ACS SASSI Option NON library of hysteretic models. Additional details on the BBC generation and the use of Cheng-Mertz shear hysteretic model is provided in a previous paper (Ghiocel, 2017)

The shear strain capacities of the wall panels were assumed to have lognormal probability distributions with the mean of 0.5% and c.o.v. of 35% for random variations and c.o.v. of 45% for composite variations, respectively. The lognormal reliability model was used to compute the wall shear failure probabilities (or fragilities).

Figures 15 through 18 shows some of the results obtained for the structural fragility analysis based on the physics-based probabilistic nonlinear SSI response simulations using ACS SASSI Options PRO and NON. Figure 15 shows the mean and 84% NEP shear strains computed in the external transverse wall Panel 17 based on the random variations and the composite variations, respectively. All seven seismic review levels are included in Figure 15.

Figure 16 shows the hysteretic behaviour of Panel 17 for the 0.95g seismic hazard level for the random and composite variations, respectively, for the largest (blue) and smallest (red) story drift simulations.

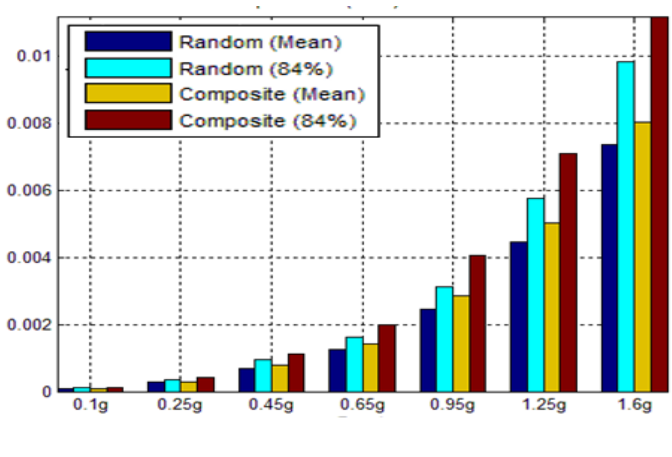
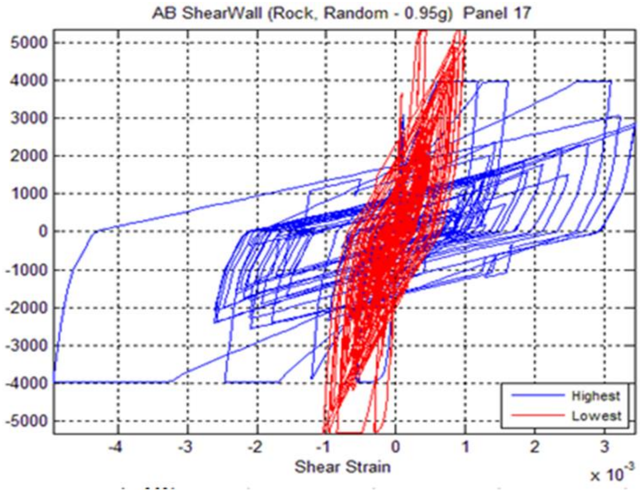
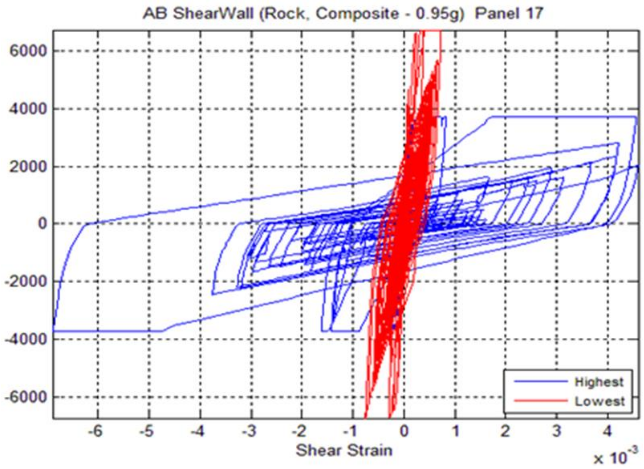


Figure 15 Wall Shear Strain Statistical Responses



a) Random Variation Inputs



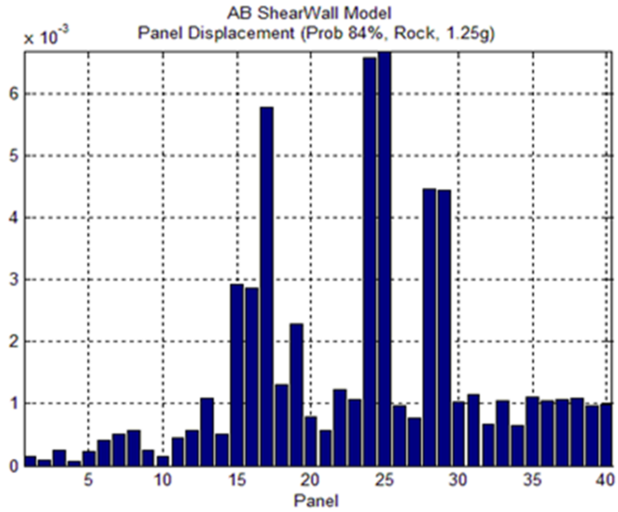
b) Composite Variation Inputs

Figure 16 Hysteretic Loops for 0.95g Review Level for Largest (blue) and Smallest (red) Simulated Response

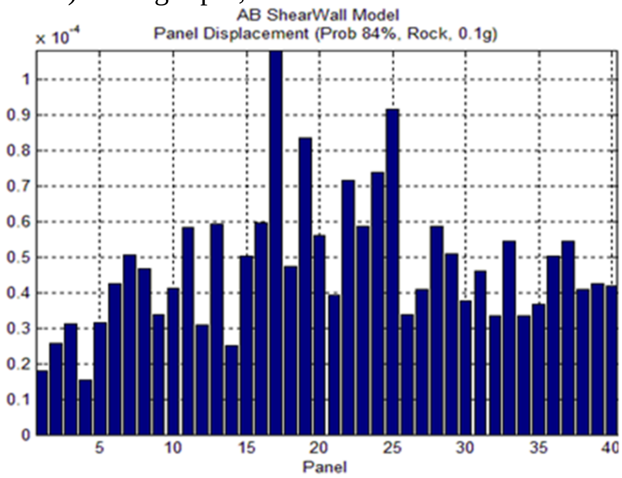
The epistemic uncertainty variations for each response quantity of interest was computed for each probabilistic simulation j by Uj = Cj/Rj, as defined in the lognormal fragility format, where Cj is the simulated response including the composite variations and Rj is the simulated response including only the random variations (Ghiocel, 2017).

Figure 17 shows the probabilistic 84% NEP story drift responses in all 40 wall panels for two seismic hazard levels, namely, the 0.10g and 1.25g. It should be noted that for the 0.10g level, the structure behaves linear elastically, and the story drift distribution across different walls is much more uniform than for the 1.25g level for which some walls behave heavily nonlinear and a result of this, the story drift distribution is highly non-uniform, in pockets of damages in few walls.

The Figure 17 results show that the use of nonlinear SSI analysis is very important to capture more realistically the dynamic behaviour the building structure for large seismic input levels.



a) 0.10g Input; Uncracked Concrete Structure



b) 1.25g Input; Highly Nonlinear Structure

Figure 17 Probabilistic 84% NEP Story Drifts in Wall Panels for 0.10g and 1.25g Review Levels

To compute the structural fragility curves for the concrete walls, the traditional lognormal format was applied. Based on the simulated probabilistic SSI responses and the probabilistic wall capacities, the conditional failure probabilities (fragility data points) were computed for all seven hazard levels using the lognormal reliability model. Then, a probability space transformation to normal space is applied to the computed probability data, and based on the linear regression in the normal space, the lognormal fragility curves were determined (Ghiocel, 2017).

Finally, the overall risks and the overall risks were computed based on the panel structural fragility curves and the seismic hazard curve both assumed to follow the lognormal distribution format, as shown in Figure 11. To compute the total risks and the overall risk, a

Monte Carlo simulation with 100,000 random samples was employed (Ghiocel, 2017).

Based on the probabilistic nonlinear SSI analysis simulations, the overall risks, Pf, were obtained for the following four scenarios:

i) Multipoint estimate approach using seven seismic hazard levels with annual probability of 3.2.e-4, 1.0e-4, 3.2e-5, 1.0e-5, 3.2e-6, 1.0e-6 and 3.2e-7

ii) Multipoint estimate approach using three seismic hazard levels with annual probability of 1.0e-4, 1.0e-5 and 1.0e-6,

iii) Point estimate approach using a single seismic hazard level with annual probability of 1.0e-4

iv) Point estimate approach using a single seismic hazard level with annual probability of 1.0e-5

Table 1 provides a comparison of the seismic overall risks, Pf, computed for the seven most affected wall panels. The predicted risk Pf are shown for two confidence levels, specifically, the mean estimates and the 90% confidence estimates.

Table 1 Overall Risk Pf Estimates for Four Seismic Hazard Level Scenarios (Seven Levels, Three Levels and Two Single Levels)

Panel #	pf mean	pf C.O.V.	pf 90%CDF
15	1.82E-007	0.71	3.60E-007
16	2.40E-007	1.06	5.48E-007
17	7.80E-007	0.49	1.26E-006
24	8.78E-007	0.60	1.55E-006
25	9.22E-007	0.68	1.71E-006
28	4.59E-007	0.24	6.04E-007
29	4.16E-007	0.46	6.65E-007

Displacement Random

Panel #	pf mean	pf C.O.V.	pf 90%CDF
15	3.94E-008	0.54	6.85E-008
16	4.34E-008	0.81	8.95E-008
17	6.67E-007	1.06	1.52E-006
24	6.56E-007	0.80	1.35E-006
25	6.17E-007	0.70	1.18E-006
28	2.98E-007	0.96	6.65E-007
29	2.30E-007	0.87	4.87E-007

Displacement Random

Panel #	pf mean	pf C.O.V.	pf 90%CDF
15	5.20E-006	2.79	1.39E-005
16	1.51E-005	4.66	2.72E-005
17	4.39E-008	1.17	1.03E-007
24	8.14E-008	3.34	2.14E-007
25	3.26E-008	2.66	8.74E-008
28	2.61E-005	3.96	4.82E-005
29	4.49E-007	4.33	8.83E-007

Displacement Random

Panel #	pf mean	pf C.O.V.	pf 90%CDF
15	1.41E-008	0.95	3.21E-008
16	2.47E-008	1.35	6.05E-008
17	3.48E-007	1.11	7.90E-007
24	2.46E-007	1.10	5.68E-007
25	2.22E-007	1.02	4.87E-007
28	7.78E-008	1.02	1.71E-007
29	4.98E-008	0.94	1.06E-007

The Table 1 results show large differences between the overall structural risks computed for the four seismic hazard scenarios. The multiple level/multipoint estimate approach provides significantly different risk predictions than the traditional one level/point estimate approach with SSI response scaling. For 90% confidence Pf estimates computed for Panels 17, 24 and 25, vary in the 1.18 1e-6 to 1.71 e-6 range for seven and three review levels, and in the 1.03 e-7 to 8.74 1-8 range for single review level. Using the single review level for the 1.e-5 annual probability is much better than using the single review level for the 1.-e4 annual probability. Anyway, using a single review level is not recommended for the future PRA reviews.

4. CONCLUSIONS

The paper illustrates the application of the probabilistic seismic SSI analysis to the nuclear structures based on the new ASCE 04-16 standard recommendations applicable to both the design-basis level (DBE) applications and the beyond design-basis level (BDBE) applications for fragility analyses.

For the DBE applications, significant differences were noted between the probabilistic SSI and deterministic SSI responses for the deeply embedded SMR structure. There is a current need for a larger number of rigorous comparative probabilistic-deterministic investigations based on the ASCE 4-16 recommendations for surface and embedded structures using detailed FE models with elastic foundations, not only sticks with rigid mats.

The ASCE 4-16 probabilistic SSI modelling including the nonlinear structure behaviour captures well the physical-behaviour of the concrete structures for the BDBE applications. The ASCE 4-16 standard provides a robust physics-based probabilistic modelling for computing the SSI response variations as a basis for predicting the seismic fragilities of the SSCs for the new design nuclear plants.

5. REFERENCES

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